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ANALYSIS OF THE SUSCEPTIBILITY OF INFILLING MASONRY WALLS TO CRACK DUE TO VERTICAL DEFORMATION OF CONCRETE STRUCTURES

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ABSTRACT

In this paper a numerical study is presented to evaluate the masonry partitions walls susceptibility to crack due to vertical deformation of concrete structures. Common solutions used in Portuguese buildings for structural systems and non-structural partition walls were analysed in this study.

The numerical study was based on a 3D FEM macro models. Two types of simulations were performed. The first type was a simulation of a masonry deep beam subjected to vertical displacements, where experimental results obtained from tests were used to calibrate a non-linear constitutive model. The second type was the simulation of a representative part of a building concrete structure, partially filled with masonry partition walls, subjected to design loadings and creep effects. The objective of these simulations was to evaluate the partition walls susceptibility to crack by modifying some properties of these walls, and also to evaluate, as close as possible, the mechanical behaviour of partition walls affected by the deformation of concrete structures with creep effects.

Keywords: Partition walls, non-structural masonry, cracking, serviceability

INTRODUCTION

Vertical movements/deformations of building structures in serviceability conditions can cause cracking in infilling masonry walls. This type of defect can impair the functionality of these walls, such as aesthetics, watertightness or acoustics and thermal behavior, and repairing is difficult and expensive to perform.

This type of structural deformations are mainly caused by vertical loading and in the case of reinforced concrete structures these deformations are considerably aggravated by the long term/creep effects. If these structural deformations are not controlled, important loading/deformations can be imposed on more sensitive adjacent parts, such as non-structural masonry partitions and enclosure walls, and damage can occur in these elements.

Masonry itself has a lower ductile capability and a lower tensile strength when compared with other building materials, therefore, masonry walls are not able to sustain significant deformations/loading without any visible damage, especially in the case of non-structural walls since these can be made with lower resistance materials. Moreover, since these sort of walls do not have any intended structural function in the building, their mechanical behavior and detailing are aspects often neglected in the design process. Taken into account that non-structural masonry is still a solution frequently used for internal or external infilling walls, it can be first concluded that this solution is highly susceptible to damage/cracking due to the movements of their support structure.

In order to reduce the damage in non-structural elements in serviceability conditions, some concrete design codes establish limits for the vertical displacement of structural elements. Despite of this fact, cracking on masonry infilling walls have been reported with some frequency in recent years, and in some cases this seem to be related with the vertical deformation of reinforced concrete beams and slabs.

In this paper a numerical study is presented to evaluate the masonry partitions walls susceptibility to crack in serviceability conditions, in particular due to vertical deformation of concrete structures. Common solutions used in Portuguese buildings for structural systems and partition walls were considered: concrete framed structure made with beams and piles and concrete slabs (beam-and-block floor system), and partition masonry walls made with horizontally perforated clay units laid with general purpose mortar.

CONSTRUCTIVE SOLUTIONS FOR PORTUGUESE BUILDINGS

Cracking in masonry walls is a pathology frequently associated to the constructive solutions used in buildings, in particular the solutions used in masonry walls and their support structures, and therefore a characterization of those constructive solutions is important.

Since the seventy’s that Portuguese buildings are made with reinforced concrete structures filled with non-structural masonry walls. In the last decade about 67% of the buildings were constructed with this constructive technology, Fig.1 (INE, 2011).

Fig.1 Example of a building being constructed with reinforced concrete structure filled with masonry walls

These reinforced concrete structures are framed made with piles and beams, and at the floor level concrete slabs made with beam-and-block floor systems are frequently used (Fig.2). The spans of slabs and beams vary between 5 to 7m, and in some case may reach higher dimensions.

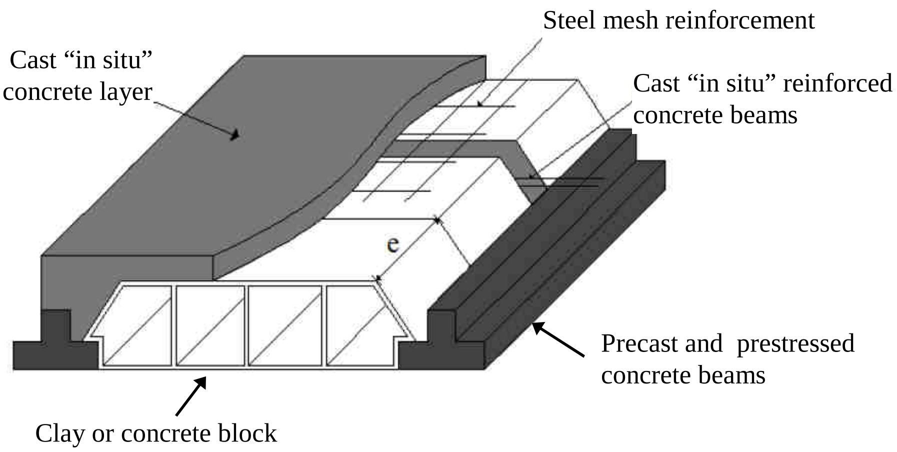


Fig.2 Example of a beam-and-block floor system

The external walls, or enclosure walls, are made with masonry cavity walls with a thickness of 30 to 35cm (without considering renderings/coatings applied on the wall), although in the last years the use of single leaf masonry walls have been growing. In general these walls include thermal insulation layers applied on the cavity of the wall or on the external face of the wall.

The internal walls, or partitions walls, are made with single leaf masonry walls with a thickness of 7cm, 11cm or 15cm (without considering renderings/coatings applied on the wall), although 11cm is the most common thickness used for this type of partitions, Fig.3.

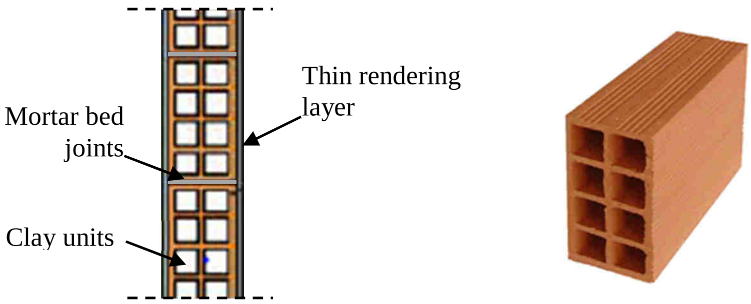


Fig.3 Example of a masonry partition wall made with horizontal perforated clay units with 11cm of thickness

Also higher thickness may be used in particular cases, such as single leaf or cavity walls between dwellings to improve certain functionality aspects (thermal/acoustic comfort, fire resistance, amongst others).

The materials most commonly used to build masonry walls in Portugal are horizontal perforated clay units laid on general purpose mortars (factory made mortars). Other type of masonry materials may also be used, although less frequently, such as lightweight or normal concrete units and lightweight mortars or special mortars for thin joints.

Partition or enclosure walls rarely include any ancillary components, such as joint reinforcement, lintels or ties. The dimensions of the walls usually vary between 3 to 6 m in length by 2,5 to 3 meters in height and for the most cases include openings for doors and windows systems. The walls are built with the units laid on vertical and horizontal mortar joints with 1 to 1,5cm of thickness. The walls are usually bonded to the surrounding structure with mortars joints executed during the erection of the walls. The connection between walls is frequently made with mortar bonding, and in some few cases with the interlocking of units.

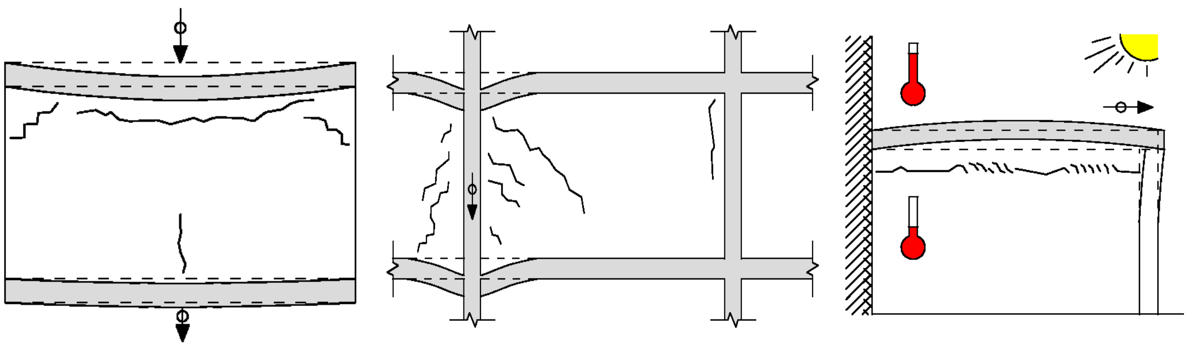
Concerning the renderings/coatings, these are frequently made with traditional factory made mortars (sand and cement or sand, cement and lime, and plaster mortars). This rendering system was applied on 88% of Portuguese buildings (INE, 2011), although in the last years other techniques have been used, in particular on the external face of the walls, such as stone cladding and external thermal insulation composite systems (ETICS). The internal renderings/coatings are also made with traditional mortars, and in partitions walls the rendering layers are usually thinner than external renderings (1 to 1,5 cm for cement based mortars and 0,5 to 1cm for plaster mortars).

CHARACTERIZATION OF CRACKING IN PARTITION WALLS

Cracking is the main pathology on partition masonry walls and applied renderings/coatings, and it has been occurring frequently in the last few years in Portugal. The main cause seems to be associated to movements/deformations of the surrounding/supporting structure of the walls, in particular concrete slabs and beams, Fig.4.

Fig.4 Examples of cracking in partitions walls due to movements of surrounding/supporting concrete structures

These structural movements/deformations can affect the partitions walls, especially if these walls are connected to the structure, and are usually caused by permanent/ live loadings, foundation settlements and thermal gradients that can occur during the life time of the building, Fig.5.



a) b) c)

Fig.5 Causes leading to cracking on partition walls: a) vertical deflections, b) foundation settlements and c) thermal movements of surrounding/support concrete structures (Sousa et al, 2015)

In the particular case of vertical deflection of beams or slabs affecting masonry partitions walls, these deformations can induce loading if the top of these walls are connected in the structure. Depending of the relative rigidity of the structure, the partition walls can act as compression members when supported on rigid elements, or act as flexural beams or deep beams, depending on the height to length ratio of the walls, if the supports are more flexible. Given the fragile nature of partition walls (lower strength and ductility when compared with its support structure), the risk for these walls to crack in serviceability conditions is high, and it is aggravated by existence of openings.

Other aspects seem can also aggravate the occurrence of cracking on masonry partition walls. During the last years the evolution of concrete technology and the use of buildings with more large and open spaces, has allowed the use of more deformable and lighter structural elements (higher spans and slenderness, flat slabs). On the other hand, the fragile mechanical behavior of partitions walls and its interaction with the surrounding structure are aspects often neglected in design. Moreover, the quality of execution of these walls it is not always the best.

LITERATURE REVIEW

There are some scientific studies and normative/technical documents with some provisions regarding the control of the vertical deflection of structural elements and detailing aspects for the execution of partitions walls in the order to avoid the damage on these walls.

Some scientific studies based on experimental tests suggests some design limits (Meyerhof, 1953; Beranek, 1987; Dias, 1994; Page,2001; Holanda et al, 2007), however it is difficult to establish from these studies a general reference since the variability of results is high. For example, limits for relative displacements of L/700 to L/3000 (being L the span of the support structure) and limits for the tensile stress in the walls of 0,1 to 0,3 N/mm^2 are found in these studies.

The Belgian National Research Institute (CSTC), based on experimental tests performed on masonry walls made with clay and concrete units, recommends that the deflection of structural elements should be lower than L/500 or L/1000, depending on the existence or not of openings (Pfeffermann, 1981).

More recently, the W023 commission for wall structures of the International Council for Research and Innovation in Building and Construction (CIB), has published a guidance manual with recommendations regarding the identification, prevention and repair of cracking on masonry walls (Sousa et al, 2015). For partition walls, CIB-W023 commission recommends deflection limits of L/500 to L/1000, depending on the existence or not of openings. However, it recognizes that these limits, or other more demanding, may not be affordable, therefore recommending complementary constructive measures to avoid cracking on partitions. Examples of such measures are the use reinforcement embedded in mortar joints or in mortar renderings/coatings, execution of provision joints on the walls, recommendations regarding the erection of the walls, amongst others aspects.

Calculation codes for reinforced concrete and masonry structures define some deflection limits in order to limit the damage in non-structural masonry partitions walls. For example, Eurocode 2 (CEN, 2004) defines a maximum deflection of L/500 for structures submitted to a quasi-permanent combination of loadings, whilst the French code (AFNOR, 2009) defines a more demanding deflection limit for structures with spans higher than 5m (L/1000 +5mm).

Calculation codes for masonry structures, such as the American code (MSJC, 2013) and the former English Code (BSI, 1995), also defines maximum deflection limits for structural masonry walls (L/600 and L/500 or 20mm) that should be verified assuming the elastic behaviour in the calculations (uncracked sections). However, in these codes it is not mentioned any admissible values for tension or compression. Nevertheless, the American code (MSJC, 2013) has section concerning specific provisions for masonry partitions, such as the establishment of minimum and maximum thickness of the wall (102 to 305mm), limiting the loading capability of the wall (e.g.vertical compression loading of 2,9kN/m and 0,2 to 0,5kN/m^2 for lateral loading), limits for the dimensions of panels and openings and support conditions of partition walls that are laterally loaded (assuming serviceability loadings and allowable stress design), definition of some constructive measures to connect/anchor partition walls to the structure and to other walls, amongst other aspects.

It is also worth mentioning that European code for masonry structures, Eurocode 6 (CEN, 2005), only provides general recommendations for structural walls and does not provide any specific limits for deflection control or constructive recommendations for structural or non-structural walls.

Finally, a French normative document (AFNOR, 2008) regarding the execution and design of non-structural masonry partition walls with thickness lower than 150mm, defines deflection limits for the support structures of these walls (the same limits mentioned by the French code for concrete structures (AFNOR, 2009)) and several constructive measures, such as maximum dimensions according to the thickness of the wall, detailing provisions for the execution of joints that disconnect the walls from the surrounding structure, connections between walls, permitted type of renderings/coatings established according to the thickness of the wall, amongst other aspects. For example, for perforated clay masonry with a gross thickness of 80 to 110mm, the maximum permitted height and length are 4m and 8m, respectively. Other dimensions may be use, however the panel should not have a surface higher than 25m^2. Moreover, the use of mortar renderings/coatings made only with cement binders is not allowed for wall thicknesses lower than 110 mm.

NUMERICAL MODEL

The numerical used in this study is based on a 3D FEM macro models, and was performed on Abaqus commercial software. Two types of simulations were made in this study:

- simulation of a masonry panel (deep beam) in order to evaluate the susceptibility to crack by modifying some properties, such as the length to height ratio, the presence of openings, and the mechanical resistance of the masonry panel;
- simulation of representative part of a building structure partially filled with masonry partition walls in order to evaluate, as close as possible, the mechanical behaviour of these walls when affected by the vertical deformations of concrete structures caused by design serviceably loading and aggravated by long term/creep effects.

a) Simulation of a masonry panel (deep beam)

This simulation consisted on a masonry panel subjected to vertical displacements applied on the top of the wall that simulates the flexural displacements of structural elements subjected to uniformly distributed loadings. Experimental data obtained from tests performed on a masonry deep beam were used to calibrate the constitutive model chosen for masonry, and afterwards this calibrated model was used to simulate a larger wall with some different properties.

The chosen constitutive model for masonry uses the concepts of damage mechanics and classical plastic theory, in particular the strain decomposition, elasticity, and the plastic flow. This model was developed by Lubliner et al (1989) for concrete, and can be applied for other brittle or quasi-brittle materials whose fracture mechanism are mainly governed by compressive crushing and tensile cracking (unreinforced or reinforced concrete, mortars, rocks). These mechanisms are implemented in the model through uniaxial constitutive laws, usually represented by tensile and compressive stress-strain relations (σt - εt and σc - εc) that can be determined experimentally from laboratory tests or from available test data. Moreover, the tensile behaviour can be also implemented through the fracture energy concept developed by Hilleborg et al (1976), by inserting in the model the values of fracture energy, G_F, and tensile strength, σ_{t0}, fig.6

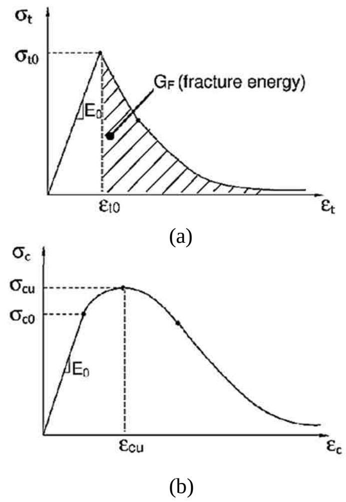


Fig.6 Generic representation of typical uniaxial stress-strain diagrams defined in the model: (a) tensile and (b) compressive behavior

The plastic strains are calculated in terms of equivalent plastic strains, which are considered in the model as tension and compression hardening variables. These variables simulate the failure mechanisms (tensile cracking and compressive crushing) and control the evolution of the failure surface. More specific details of this model and its application to masonry can be found in given literature (Lubliner et al, 1989; Sousa et al, 2013; Sousa et al, 2015).

The experimental data used for calibration of the constitutive model was obtained from a flexural test performed on a double supported masonry panel, made with horizontal perforated clay units laid on general purpose mortar joints (Pereira, 2005). The panel had the dimensions of 2m x 4m x 0,11m (height x length x thickness) and no renderings/coatings were applied. A vertical load was uniformly distributed along the top of the panel trough a reaction beam. The loads and vertical displacements at middle span were measured until the maximum/breaking load, with a loading cell and displacement transducers. The values reported in this experiment for maximum/peak load and middle span deflection were 55,2kN and 1,87mm. The fracture mechanisms reported were a vertical crack near the middle span of the wall, followed by a local crushing near the corner supports of the wall, fig.6.

This test was simulated by computer in order to calibrate the constitutive model of masonry. The mechanical properties and constitutive laws of masonry needed for the constitutive model were estimated from experimental data available in literature (Mojsilovic 2011; Pereira 2010; Pereira, 2005; Pluijm 1997; Lourenço, 1996; Carvalho,1990), table 1.

Table 1 Mechanical properties used for the constitutive model of masonry

E_0 Elasticity modulus (N/mm^2)	ν Poisson Coef. (-)	σ_{cu} Compressive strength (N/mm^2)	σ_{to} Tensile strength (N/mm^2)	G_F Fracture energy (Nm/mm^2)	ψ Dilatation angle (º)
2000	0.1	1.3	0.4	0.015	11

The complete mechanical response of the masonry panel, i.e. force-displacement curve F-δ measured at middle span until maximum load, was used in order adjust the numerical results to the experimental data. By changing some specific parameters on the constitutive model, the best adjustment obtained between numerical and experimental results gave a difference of 1% for maximum/peak load and 15% for ¼ of the peak load. Also the potential fracture pattern obtained from the simulations (given by the minimum principal plastic strains just after the peak load) is similar to the fracture pattern observed in the experimental test, fig6-c. These results were considered accurate enough, considering the variability of results usually accepted for masonry tests performed in laboratory conditions (variation coefficient between 10 to 20% for the strength properties (CEN, 1998; CEN 1999)).

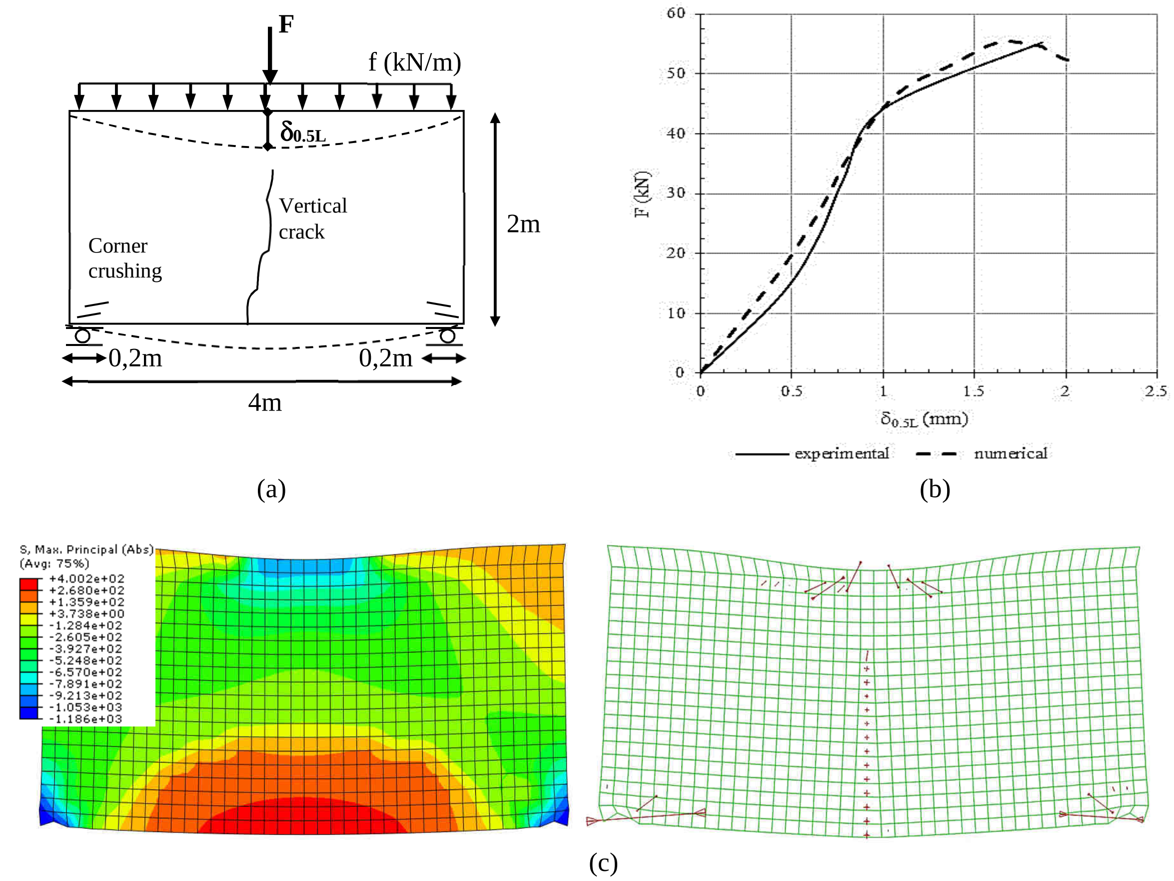


Fig.6 Calibration of the masonry wall constitutive model: (a) flexural test performed and fracture pattern; (b) best adjustment obtained for the mechanical response of the wall; (c) principal stresses distribution (in kPa) at peak load and minimum plastic strain vectors just after the peak load;

After calibrating the numerical model, a reference masonry panel, made with same materials as the test sample referred before, was chosen with higher dimensions in order to represent the dimensions more commonly used for this type of masonry partition wall. The chosen dimensions were 2,75x 5,5x 0,11m (height x length x thickness). Moreover, the mechanical contribution of renderings/coatings was not considered in this reference situation, since it was assumed the existence of a thin layer of plaster mortar on both sides of the wall (most common situation for partition walls).

The structural model used for this reference situation was similar to the test wall sample, fig.7. The loading was simulated by imposing a generic displacement law function on the top of the wall, δ(x):

$$\delta_{(x)} = 16\,(5kL^3)^{-1}\,(x^3-2Lx^2+L^3)\,x \qquad (1)$$

L= span of the wall assumed equal to the span of the structure

k= constant regarding the relative deflection of the structure (e.g. 500 to 1000)

x= considered position on the top of the wall (between 0 an L)

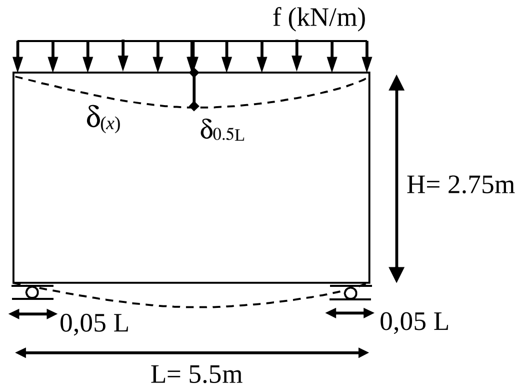


Fig.7 Structural model used for the simulation of the reference masonry partition wall

Equation (1) was deduced from flexural elements subjected to evenly distributed loads in elastic regime and simply support conditions. This law simulates the deflection of structural beams or slabs affecting the partitions, assuming a full contact and perfect bonding conditions between the top of the wall and the structure.

To evaluate the susceptibility of this particular case of masonry partition wall to crack, some changes in the geometry and in the mechanical characteristics were made in the reference masonry panel. Each one of these changes were simulated separately, therefore keeping the other characteristics equal to those of the reference situation. The variations introduced in the reference panel were:

- panels with ddifferent length to height ratios by shortening of the length (L/H =0.5, L/H =0.75, L/H=1)
- panel with a central or lateral door opening (1,9 x0.9m^2)

- panel with a stronger rendering/coating layer applied on both sides of the wall (cement based mortar with 2cm of thickness and resistance class M5);
- local reinforcement ($0.4x0.6m^2$) near the 2 supports of the panel with stronger materials in order to reduce the corner crushing reported in the experiments (e.g. stronger masonry or filling the units voids with M5 Mortar).

Force-displacement curves at middle span, f - $\delta_{0.5L}$, was used to evaluate the mechanical response for these different situations.

The mechanical properties of stronger rendering/coating layer such as modulus of elasticity, tensile and compressive strengths, fracture energy, and the uniaxial laws stress-strain in compression and tension were estimated from experimental data obtained from literature (Veiga, 1997) and from Model Code for concrete structures (CEB-FIP, 2010; CEB-FIP, 1990). A more detail explanation about how this properties were estimated can be found in given literature (Sousa et al, 2013; Sousa et al, 2015).

b) Simulation of a representative part of a building structure filled with masonry partitions

A 3D framed concrete structure was designed according to Eurocode 2 (CEN, 2004) to verify the ultimate limit and the serviceability limit states, in particular the deformation control measures of concrete structures to avoid the damage in non-structural elements (maximum relative displacements of slabs and beams lower than L/500 for a quasi-permanent combination of actions/loads, including recommended span/depth ratios). The concrete structure was considered to behave in a linear elastic regime, without the contribution of steel reinforcement, since it is expected low tensile and compressive stresses for serviceability loadings (uncracked sections). The building structure had the following generic characteristics:

- building with 3 floors (2 floors for resident or office use and one floor with an inaccessible use/horizontal roof) with 2,75m of height (measured from the floor to ceiling);
- loading conditions:
  - live loads on floors: $\Sigma Q = 2kN/m^2$ (residential use); $\Sigma Q = 3kN/m^2$ (office use); $\Sigma Q = 1\ kN/m^2$ (floor with inaccessible use)
  - permanent loads (Self weight) - ΣG: piles and beams ($25kN/m^3$); slabs ($3,7kN/m^2$); partitions ($1kN/m^2$); floor and wall renderings/coatings ($1,5kN/m^2$);
  - combination of loads (quasi-permanent): $\Sigma G + 0.3\Sigma Q$
- piles and beams made with a C25/30 concrete resistance class;
- piles and beams cross sections: 0,3x0,3m and 0,3x0,5m, respectively;
- effective spans for beams and slabs: 5.8m;
- concrete slabs: beam-block floor system with 0,25m of thickness;
- constitution of the masonry partition wall: panel with 2,75x 5,5x 0,11m (height x length x thickness), made with horizontal perforated clay units laid in general purpose mortar, coated with a thin layer of plaster on both sides;

In order to reduce the computation effort, only a central part of the building structure was simulated by considering symmetry conditions on the concrete structure in XY directions, fig.8.

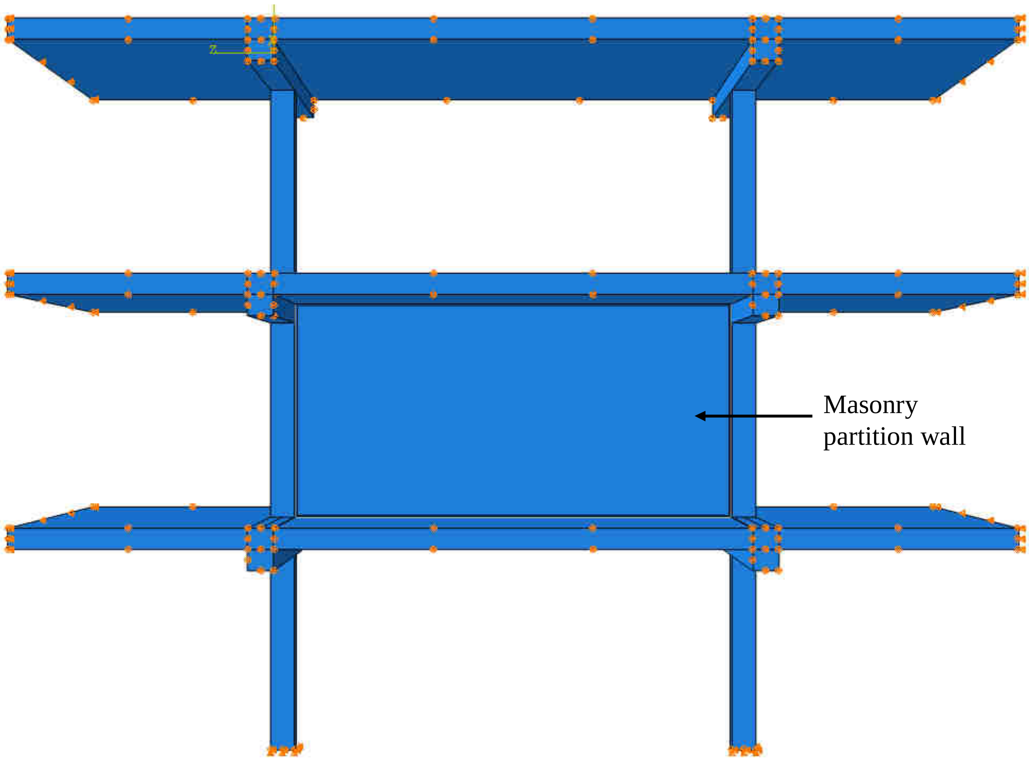


Fig.8 Symmetry conditions for the geometrical model used for the simulation of the concrete structure filled with a masonry partition wall

The same reference clay masonry wall (2,75x 5,5x 0,11m) and the same non-linear model used in the first simulation (see a)) was used to simulate the partition wall. The connection of this partition was considered bonded to the concrete structure by mortar joints, however simulated in two different conditions:

- by top/bottom mortar joints, and a clearance of 1 cm in both lateral sides of the panel to avoid contact with the piles;
- by one bottom and two lateral mortar joints, and a clearance of 1cm on the top of wall to avoid contact with the concrete slab.

The creep effects on the concrete structure was considered according to Model code 2010 methodology (CEB-FIP, 2010). This effect is considered to affect only the strains, and produce a viscoelastic behaviour on concrete for stress levels lower than 40% of the compressive strength or lower than the tensile strength. Considering that loading of the structure will take place at 28 days of age (t0 = 28days), the total strain of concrete at a considered age, $\varepsilon_{T(t)}$, can be determined according to following expression (CEB-FIP, 2010):

$$\varepsilon_{T(t)} = \varepsilon_{i(t0)} + \varepsilon_{c(t)} = \sigma_{(t0)}/E_{(t0)} \left[\sigma_{(t0)}/\sigma_{(t)} + \varphi_{(t0,t)}\right] \quad (2)$$

$\varepsilon_{i(t0)}$ - initial strains at the age of loading (considered at 28 days)

$\varepsilon_{c(t)}$ - creep strains at a considered age, determined from: $\sigma_{(t0)}/E_{ci}..\varphi_{(t0,t)}$ (E_{ci} -modulus of elasticity at the age of 28days)

$\sigma_{(t0)}$ / $\sigma_{(t)}$ - Ratio between initial stress level at the age of loading to stress at a considered age

$E_{(t0)}$ - Modulus of elasticity at the age of loading (considered at 28 days)

$\varphi_{(t0,t)}$ - Creep coefficient at a considered age

The considered age, t, for this numerical analysis was 50 years (service life for ordinary building structures), considering indoor atmospheric conditions (R.H.= 60%). Nevertheless, it is highlighted that the most part of the creep effects on the structural deflections will take place in the first years of age of the concrete structure, fig.9, therefore the damage inflicted on partitions by creep effects may occur before the age considered.

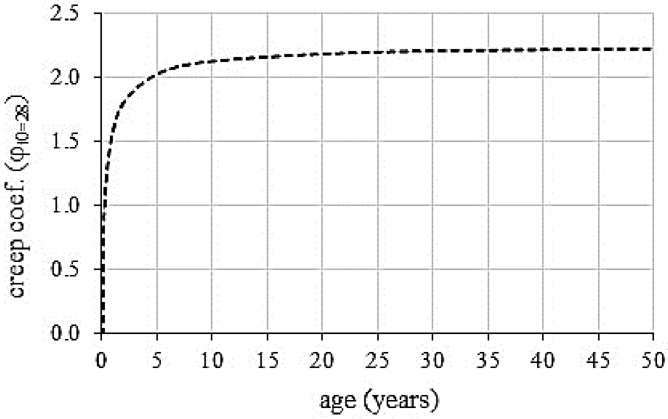


Fig.9 Evolution of the creep coefficient throughout time for the concrete beams used in simulations (CEB-FIP, 2010)

Creep effects on the masonry partition was implemented in a similar way by using an average value for the creep coefficient calculated from Eurocode 6 (CEN, 2005) long term creep coefficients defined for clay masonry (infinite time creep coefficient, $\varphi_{(t0,\infty)}$, varies from 0.5 to 1.5). The creep strains, calculated from $\sigma_{(t0)}/E_{ci}..\varphi_{(t0,\infty)}$, were added to the initial strains defined for the uniaxial constitutive laws of masonry, fig.10.

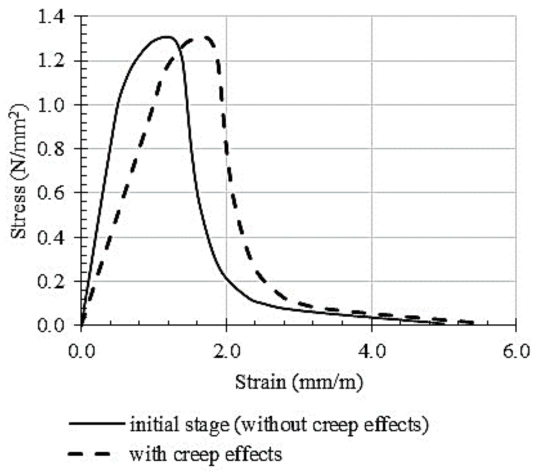


Fig.10 Uniaxial compression constitutive law for masonry, with and without the creep effects

Regarding the loading conditions, considering that the partitions are built after the casting off the concrete structure (after 1 month of curing), it was assumed that the initial deformation of the structure imposed by its self-weigh and self-weigh of partitions will not affect the partition wall, therefore only the initial deformation imposed by the self-weigh of the renderings/coatings will affect the partition. Nevertheless, all the loads will affect the partition when considering the long term creep effects. This principle was implemented in the model by an equivalent long term modulus of elasticity for the concrete structure, calculated through equation (2) and valid only for the analysis of the masonry partition.

As mentioned before, contour joints were created in the panel in order to simulate the bonding between the concrete structure and the masonry panel. This bonding was simulated by a Mohr-Coulomb friction model and by mortar joints whose behaviour is governed by the constitutive model developed by Lubliner et al (1989). The mechanical parameters and uniaxial laws needed for constitutive model of the mortar joint were determined from experimental data available from literature (Veiga, 1997) and from Model Code (CEB-FIP, 2010). In the particular case of the tensile behaviour, the experimental results obtained by Pluijm (1997) were used to estimate the bonding behaviour between masonry and mortar.

The mechanical characteristics of the concrete structure, masonry partitions walls and bonding joints are presented in table 2.

Table 2 Characteristics of concrete structure, masonry panel and interface joint used in the simulations (average values)

Components	φ Creep coef. (-)	$E_{(t0)}$ Elasticity modulus (N/mm^2)	ν Poisson Coef. (-)	σ_{cu} Compressive strength (N/mm^2)	σ_{to} Tensile strength (N/mm^2)	G_f Fracture energy (Nmm/mm^2)	ψ Dilatation angle (º)	μ Friction coef. (-)
Concrete Piles and beams	2,2	31000	0.20	33	2.6	-	-	-
Concrete slabs (beam and block floor system) (1)	2,2	17000	0.20	33	3.2 (2)	-	-	-
Masonry panel	1	2000	0.10	1.3	0.4	0.015	11	-
Mortar M5 (bonding joints)	0	7200	0.15	5.0	0.3 (3)	0.012	35	0.4

(1) values deduced from technical documents approved by a Portuguese government institution (LNEC, 2012; LNEC, 2014)
(2) Limit tensile stress for uncracked sections (elastic regime)
(3) Tensile bonding strength determined from experimental data (Pluijm, 1997)

RESULTS AND DISCUSSION

a) Simulation of a masonry panel (deep beam)

The results obtained from the numerical simulations are presented in fig.11 (load capacity, f, versus displacement at middle span, $\delta_{0.5L}$, both calculated per wall length) and in table 3 (maximum load capacity, f_{max}, and corresponding displacements at middle span of the wall, $\delta_{0.5L}$).

From the results obtained, the following aspects are highlighted for the type of masonry partition in analysis:

- corner reinforcement can significantly increase the deformation and strength capability of the panel;
- the use of M5 mortar coatings can significantly increase the strength capability of the panel;
- openings can significantly reduce the loading capability of the panel, especially lateral openings
- shorter span panels have better deformation and strength capability;
- relative displacements values without considering creep effects on masonry, are around 1/3000 to 1/4000, which indicates a very low deformation capability, therefore a high susceptibility to crack if affected by structural deformations.

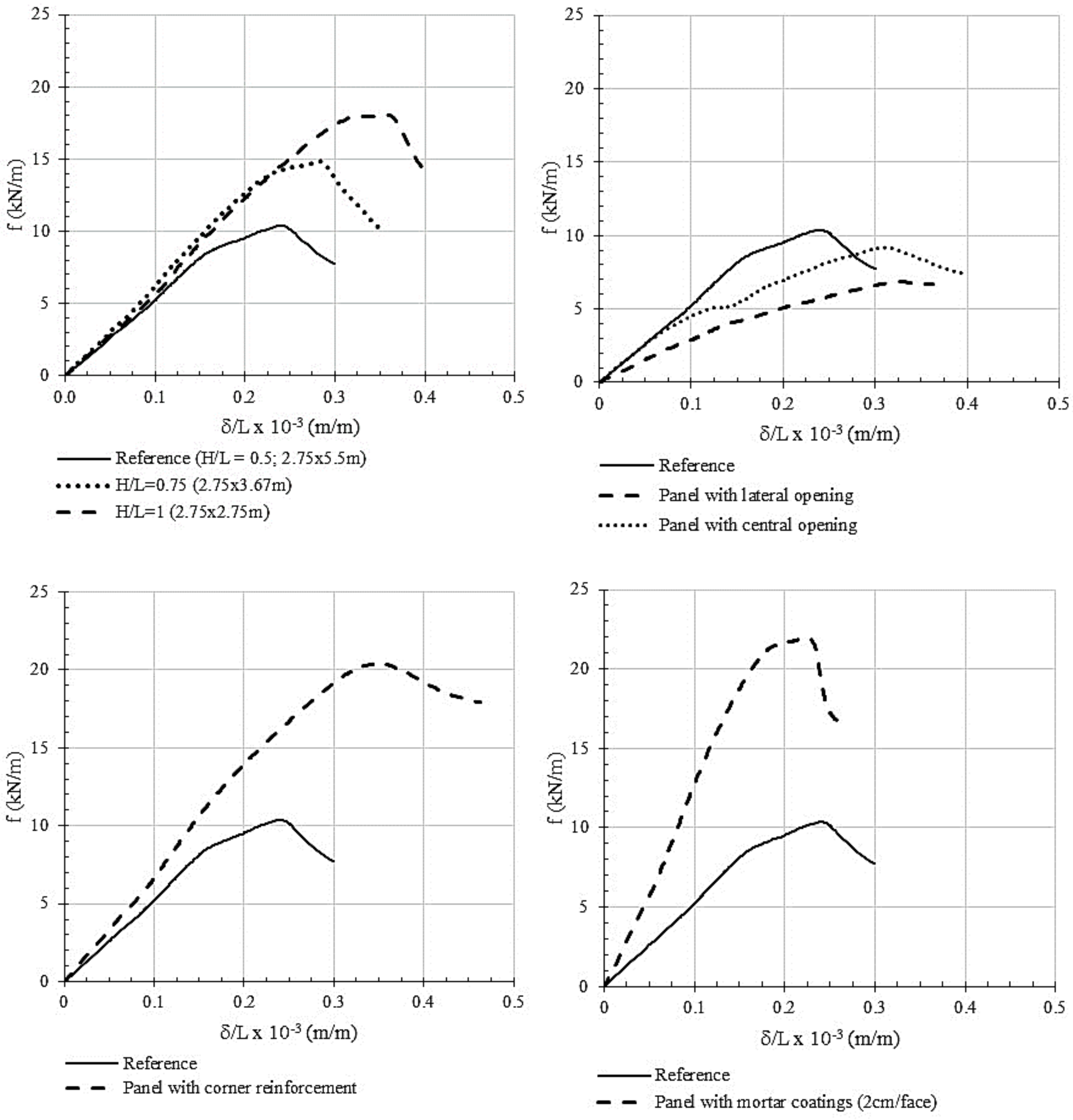


Fig.11 Mechanical response obtained from simulations for masonry panels with different characteristics

Table 3 Numerical results for masonry panel with different characteristics

Situation	$\mathbf{f_{max}}$ (kN/m)	$\boldsymbol{\delta_{0.5L}}$ (mm)	$\boldsymbol{\delta_{0.5L}}$ **/ L** (-)
Reference panel : H/L=0.5; without corner reinforcement, strong mortar coatings, or openings	10.4	1.31	1/4196
Panel with H/L=0.75	14.8	1.04	1/3538
Panel with H/L=1	18.0	0.99	1/2792
Panel with corner reinforcement	20.4	1.93	1/2853
Panel with stronger mortar coatings	22.0	1.25	1/4409
Panel with lateral opening	6.9	1.81	1/3041
Panel with central opening	9.2	1.72	1/3198

b) Simulation of a representative part of a building structure filled with masonry partition

The maximum values for vertical deflections and principal stresses obtain for the structure in serviceability conditions (residential and office use), considering the creep effects, were:

- vertical deflection for beams: 3,7 to 5,2mm (middle span);
- vertical deflection for slabs: 7,3 to 8,3mm (middle span);
- tensile and compressive stresses for beams: ±1,7 to 2,1 N/mm^2 (middle span);
- tensile and compressive stresses for slabs: ±1,1 to 1,5 N/mm^2 (near the beams);
- compressive stresses for piles: -7,5 to -8 N/mm^2.

The maximum values for vertical deflections and principal stresses obtain for the masonry panel were:

- tensile/compressive stresses (panel with top/bottom bonded joints and lateral clearance): +0,3 to +0,4MPa (lower middle span) / -0,85 to -0,92 MPa (near corner supports);
- tensile/compressive stresses (panel with bottom/lateral bonded joints and top clearance): +0.25 to +0.27 (lower middle span) / -0.54 to -0.57 (near corner supports);
- vertical deflection (panel with top/bottom bonded joints and lateral clearance): 3,3 to 3,5mm (top middle span of the wall);
- vertical deflection (panel with bottom/lateral bonded joints and top clearance): 2,5 to 2,7mm (lower middle span of the wall).

From the results obtained, the following aspects are highlighted:

- the overall maximum tensile stresses obtain for the structural elements are in the range of elastic regime (uncracked sections), and the maximum deflections for slabs and beams, considering the creep effects after 50years, are lower than L/500 for residential and office use (L/700 to L/800 for slabs without the influence of the partition; L/1000 to L/1600 for beams), fig.12;

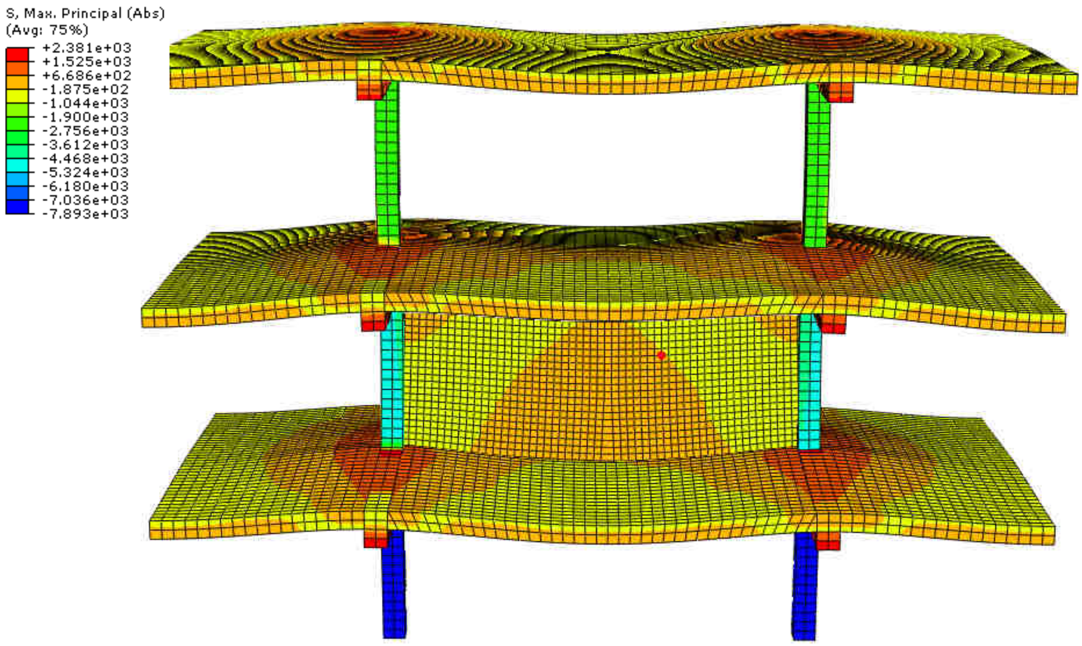


Fig.12 Example of the principal stresses (in kPa) and deflections for an office building

- the partition panel with top/bottom mortar joints and lateral clearance joints is affected by a top defection near L/1700 and to a maximum tensile stress close or equal to the tensile strength of masonry, especially for the case of office building, where cracks in the middle span of the wall will arise, fig.13;

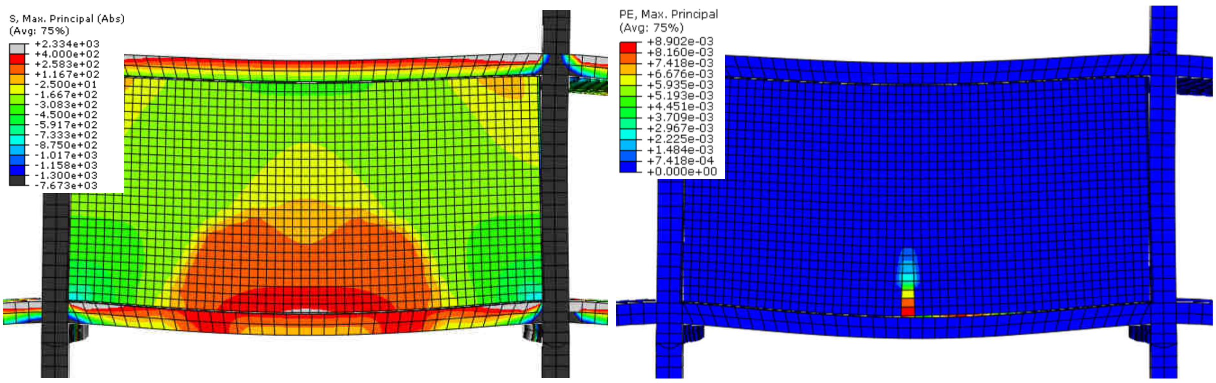


Fig.13 Detail view of the principal stresses (in kPa) and plastic strains indicating cracks in the middle span of the partition panel built in an office building

- the partition panel with bottom/lateral mortar joints and top clearance a joint is affected by a bottom deflection near L/2200 and to a maximum tensile stress of about 2/3 of tensile strength of masonry, either for residential or office use, without any cracks on the wall, fig.14.

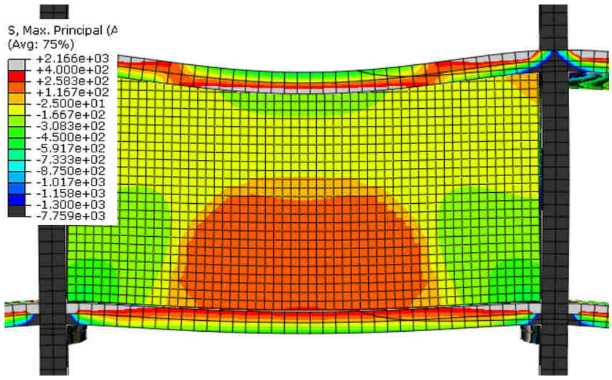


Fig.14 Detail view of the principal stresses (in kPa) in the partition panel built in an office building

CONCLUSION

This study shows that masonry partitions walls, built with 11 cm horizontal perforated clay units, are susceptible to crack when affected by the structural deformations aggravated by creep effects in serviceability conditions, especially if the panels are bonded to the concrete structure by top and bottom mortar joints, since the panels are more susceptible to the structural deformations.

Moreover, given the fragile nature of these type of masonry partitions, it seems very difficult to accomplish the structural deformation limits established in the design codes to avoid damage in partition walls (L/500 to L/1000). Therefore, the use of reinforcement techniques, provision joints or lower deformation limits is recommended for this type of masonry.

Regarding the use of reinforcements, steel reinforcement in the mortar joints and reinforced mortar coatings are the most common techniques.

Lower deformation limits for the structure can be used if affordable, and it should be evaluated case by case. From this case study, it can be concluded that displacements on the top of the wall affected by creep effects should be lower than L/1700, highlighting the fact that other parts of the structure without the influence of the partitions have higher displacements, although lower than the limits established in the design codes.

Alternatively, the use of top joints made on the top of the wall, with enough clearance to avoid contact caused by structural deformations, can reduce the susceptibility for cracking to occur in the partitions. Therefore it is recommended the use of these joints filled with a material with a very low structural resistance.

In any case, it should be considered that the presence of openings on the partition walls will significantly increase the susceptibility to crack, while the use of stronger rendering systems or corner reinforcement can decrease it.

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